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GRANBY PUMPING PLANT FOUNDATIONS AND DESIGN

by W. R. Judd, A.M. ASCE
and W. H. Wolf

CONSTRUCTION DIVISION
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GRANBY PUMPING PLANT FOUNDATIONS AND DESIGN

W. R. Judd[1] and W. H. Wolf[2]

FOREWORD

The foundation conditions at the Granby Pumping Plant, their relation to the design, and the unusual design analyses of this structure are described herein.

INTRODUCTION

Granby Pumping Plant is a major feature of the Colorado-Big Thompson Project (Figure 1). The project is a transmountain diversion development which takes waters from Grand Lake, the headwaters of the Colorado River, on the western slope of the Colorado Rockies, and by means of the Adams Tunnel through the Continental Divide, diverts this water to the Big Thompson River on the eastern slope. To supplement the water supply available from the drainage area above Grand Lake, water from tributaries below the lake is collected on the western slope by Granby and Willow Creek Reservoirs and pumped to Shadow Mountain Reservoir. From this point, gravity flow carries the water through Grand Lake, the Adams Tunnel, and to the eastern slope where it is stored for later release as a supplemental supply for the irrigation of 708,000 acres of fertile farm land in the Platte Valley. On the eastern slope, the natural drop of the water, about 2,800 feet between the outlet of the tunnel and the foothills storage reservoirs, is utilized in the generation of power. Granby Pumping Plant is located on the north shore of Granby Reservoir. It lifts the water from the reservoir to the head of a 1,100-cfs canal which conveys the water to Shadow Mountain Reservoir.

The elevation of the reservoirs on the western slope can be controlled by remote control from the pumping plant. Diversion through the Adams Tunnel can be regulated, and telemetering equipment housed in the plant keeps operators informed concerning the elevation of the various reservoirs and their discharges (Figure 2). The pumping plant is an unusual structure being the equivalent of a 15-story reinforced concrete building 187 feet in height, 133 feet of which, however, are underground.

FOUNDATION CONDITIONS

Granby Reservoir lies in a natural valley created by glaciers moving westward from the Front Range of the Rocky Mountains. The glaciers in their downward sweep from the mountains carved new valleys in the soft beds of siltstone and sandstone which had been deposited in Tertiary times, leaving

1. Geologist, Engineering Geology Branch, Design and Construction Division, Bureau of Reclamation, Denver, Colorado.

2. Engineer, Structural and Architectural Branch, Design and Construction Division, Bureau of Reclamation, Denver, Colorado.

in their wake a mass of glacial debris composed of a heterogeneous mixture of sand, gravel, boulders, and silt. After the glaciers had retreated, the present drainage patterns of the country were formed. The path of a tributary glacier then became the present valley of the Colorado River in the vicinity of Granby Reservoir.

In the present area of the reservoir, recent alluvial materials overlying glacial debris have been found, which, in turn, overlie siltstones and soft sandstones near the top of the Tertiary North Park formation. This formation also has been injected with lava flows which, however, were not encountered at the structure site. The reservoir lies in a broad U-shaped valley with steep walls. These walls have numerous exposures of the basement rock--granite and granite gneiss.

Thus, the broad U-shaped valley provided an excellent reservoir site in which sufficient water could be impounded by constructing Granby Dam and four small dikes. The next problem was to move water from the reservoir up to Shadow Mountain Reservoir and Grand Lake, which were 85 feet higher at maximum high water in Granby to 180 feet at low water (Figure 3). Numerous sites for the pumping plant necessary for such a scheme were investigated. After brief studies, many sites were eliminated due to obvious construction difficulties or exceedingly high costs. Intensive studies were made on what appeared to be four comparable schemes called A-1, A-2, B, and C. The A-1 and A-2 schemes involved 3,000 feet of discharge conduit, 2 miles of canal at the northwest end of Granby Lake, and open excavation and backfill. The A-1 scheme involved three surge suppressors and three 7-foot 3-inch reinforced-concrete discharge conduits. The embankment to fill this cut would have been raised 7 feet above the maximum high-water surface. The A-2 scheme also involved the construction of a valve house with three slide valves and a larger plant to house the discharge valves. With this scheme, one surge tank was utilized, with one 11-foot-diameter discharge conduit.

Scheme B necessitated the driving of an 11-foot intake tunnel, a vertical stope to the surface of the plant site, and construction of a 73-foot-diameter circular building, using the face of the excavation for the exterior form.

Under Scheme C the pumping plant was located some distance down the reservoir rim, and the plant would have been placed in granite. A tunnel intake and discharge would have been required, as well as a 6- to 7-mile sidehill canal. Such a canal at this elevation and in this climate would have involved considerable difficulty with freezing and sidehill drainage.

The costs, operation, and construction of these schemes were analyzed. Scheme A-1 with modifications (such as the elimination of the surge tank) was selected because it would be possible to (1) take any one of the three units out of service; (2) it involved only open-cut construction which would permit simultaneous operations; (3) both the construction and operating hazards were minimized; and (4) the rectangular plant with semicircular ends was relatively more conventional, and thus offered better operation facilities.

FOUNDATION EXPLORATION

Exploration of the foundation by drilling, test pitting, and trenching revealed the presence of a high-water table, and that the primary foundation material would be the siltstones and sandstones of the North Park formation. The siltstone is friable, moderately cemented, light gray to reddish buff and brown in color. Severe fracturing was noted near the surface of the beds, but the material near the final pumping plant grade became more massive,

although there were a few thin laminae and partings of clay. The cementing material was silt and clay which led to the inference that construction difficulty would be encountered if the excavation became wet. The siltstones were found to occupy a position intermediate between soils and rock, and, therefore, exhibited characteristics of both. Saturation of the siltstone in the laboratory caused dilation and subsequent increase in void ratio and decrease in strength and density. These values were probably lower than those inherent in the material in its original state where saturation took place under the consolidation and restraint imposed by the overlying beds. The laboratory tests indicated, however, that swelling and spalling during excavation could be expected.

The siltstone was tested in the laboratory for permeability, compressibility, elasticity, and for durability under conditions of wetting and drying. The samples yielded inelastically under the relatively low stresses of 100 psi. The compression tests were quite variable, the wet strength ranging from 200 to 1,240 psi and the dry strength varying from 1,320 to 5,000 psi. Sonic modulus of elasticity tests on the wet samples yielded values of E ranging from 167,000 to 454,000. Static modulus tests, also conducted, indicated that the moduli of elasticity ranged from 0 to 2,600,000. The latter value was the exception as most of the samples were so soft that accurate test results could not be obtained. The permeability was sufficiently low that little trouble from water inflow was expected in the excavation.

Triaxial shear tests on a recompacted sample of soft, reddish-brown siltstone having an average dry density of 109 lb per cu ft and a moisture content of 19 percent gave a tan Ø of 0.62 and a cohesion of 9.3 psi; a sample of very hard, bluish-gray siltstone, with an average density of 129 lb per cu ft and a moisture content of 5 percent, after recompaction, gave a tan Ø of 0.86 and a cohesion of 26.0 psi. The specific gravity of both samples was 2.65. The samples were recompacted as the undisturbed siltstone was too hard to prepare by standard procedures. The relationship between the laboratory shear values of the remolded samples and those that actually existed in the field was somewhat speculative. However, it was assumed that whereas the laboratory cohesion values might be considerably different from the in-place values, the laboratory friction values would be acceptable. As a result of these studies it was decided that the 6.25 tons per square foot design load was permissible for the siltstone.

Geologic study disclosed minor slumping in the natural 3:1 to 4:1 valley slopes. These slumps occurred at points of oversteepening and at groundwater seeps. It was thought that the valley sides might heave and flow during reservoir inundation and thus close the inlet channel, and that protection should be provided to prevent leveling of the natural slopes and erosion due to wave action. In view of the instability, the decision was made to grade an adjacent hill to a stable slope and to place riprap wherever the bare siltstone would be exposed to wave action. It was further decided that the excavation slopes should be about 1:1 during construction, and the embankment on the intake side of the plant would have 4:1 slopes to match the adjacent topography and be riprapped to prevent erosion.

(Figure 4) An exploratory trench was excavated along the pumping plant site. The siltstone appeared quite hard and durable indicating that blasting would possibly be required for excavation. But during an inspection of the trench, a heavy rain occurred which softened the siltstone in the bottom of the trench, so that wherever it was walked upon it became clayey and soft.

CONSTRUCTION DIFFICULTIES DUE TO GEOLOGIC CONDITIONS

From the outset, concern was expressed that the slopes planned for the excavation would have to be very flat for stability. (Figure 5) Laboratory tests indicated, however, that the 1:1 slope should be stable. It was recommended that a 10-foot berm be excavated on top of the siltstone, and the overlying glacial gravels be cut back on about a 1-1/2:1 slope from the berm. This berm was omitted, however, during initial construction operations until it was found that continual raveling of the glacial overburden material endangered the workmen in the cut below; the berm was then constructed as originally planned and further troubles from this cause were not experienced.

During excavation, faults were exposed in the walls. There was a severe fault on both the left and right excavation walls, as illustrated. It was concluded that these faults could cause some slides and result in overexcavation, but it was not expected that major damage would occur. (Figures 6, 7) This was later verified when three slides occurred on the left-hand side. They resulted in no damage but required overexcavation. All three slides were due to the existence of the fault zones and lubrication from ground water.

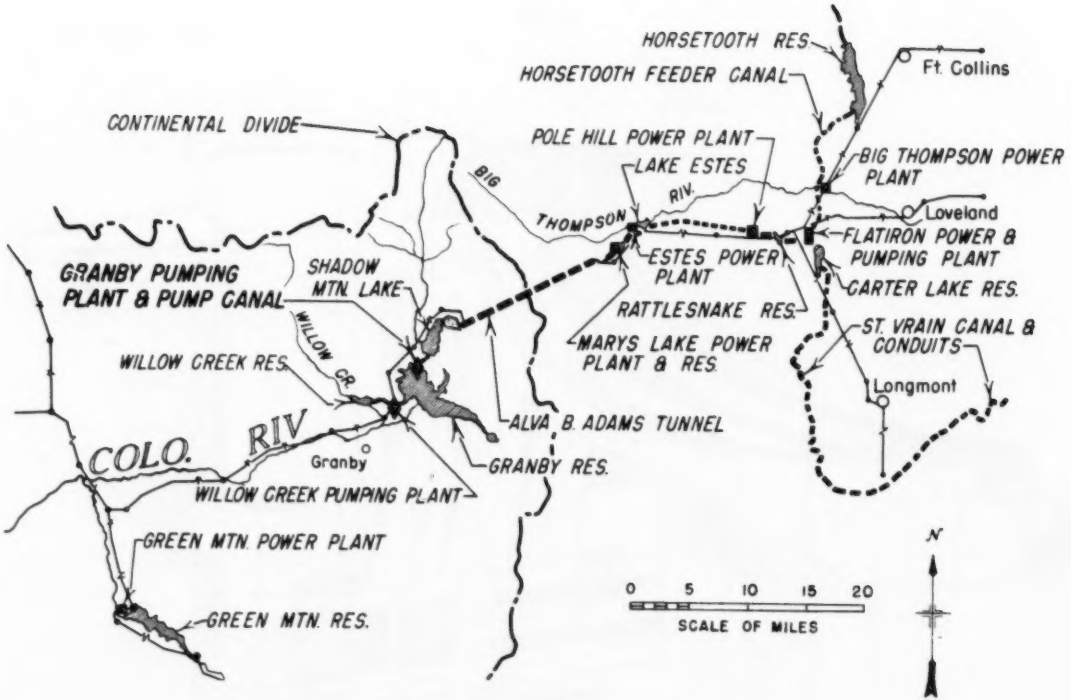
It was also found that the siltstone was quite susceptible to deterioration by moisture because of the silt-clay cementing material. (Figure 8) If water was allowed to collect on the floor of the excavation, and subsequently trucks and other equipment worked upon this wet surface, the siltstone was turned into a mud slurry which soon became impassable. It became necessary, therefore, to keep the floor of the excavation well drained to permit mobility of the excavation equipment. This difficulty also verified earlier conclusions that the newly exposed surfaces of the siltstone at final grade should be protected with some type of lining to prevent spalling and deterioration before placement of the concrete. Thus, when final grade was achieved, the walls and floor were coated with asphalt emulsion. This treatment was only partially successful, as the emulsion was placed after some spalling had occurred thus preventing a good bond with the rock.

FINAL DESIGN OF THE PLANT

General Arrangement

(Figure 9) The pumping plant structure contains three pumping units. Each unit consists of a single-stage, vertical-shaft, centrifugal pump which discharges 200 cfs at a rated head of 186 feet at 327 rpm. The pumps are driven by 6,600-volt, 3-phase, 60-cycle, 6,000-horsepower motors, and each unit is provided with hydraulically operated butterfly valves on the intake and discharge sides so the unit can be inspected and repaired without interfering with the operation of the other units.

The intake structure is semicircular in plan with provision for three 7-foot 3-inch-inside-diameter concrete conduits. There are no emergency gates at the intake structure, but there is provision for placing a bulkhead to close the entrance to each of the three intake pipes. The intake conduits are of reinforced concrete with wall thicknesses increasing from 18 inches near the intake to 27 inches under the maximum fill near the plant. The three discharge conduits are 87-inch-inside-diameter steel pipes encased in concrete. These discharge into an 11-foot-inside-diameter pressure conduit of reinforced concrete leading to the open canal section which carries the water to the Shadow Mountain Reservoir.



COLORADO - BIG THOMPSON PROJECT

Fig. 1

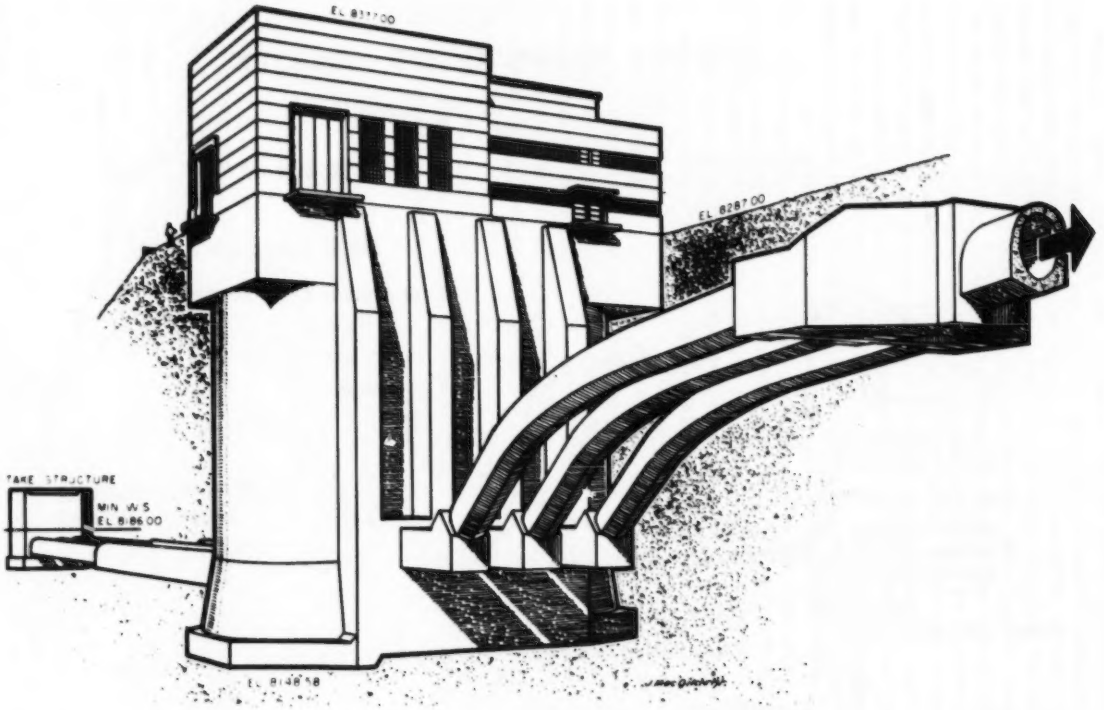
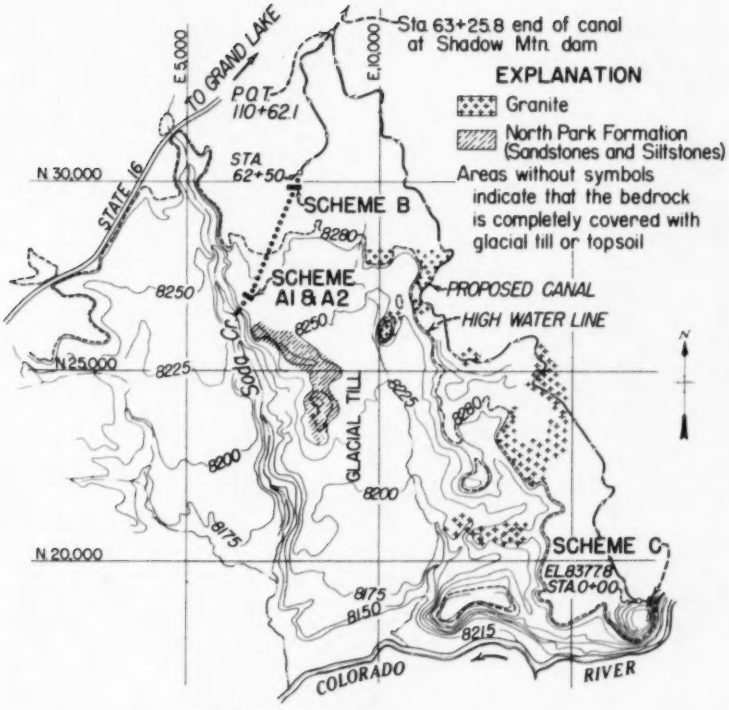


Fig. 2



GRANBY PUMPING PLANT
GEOLOGIC MAP

Fig. 3

Fig. 4

Fig. 5

Fig. 6

Fig. 7

Fig. 8

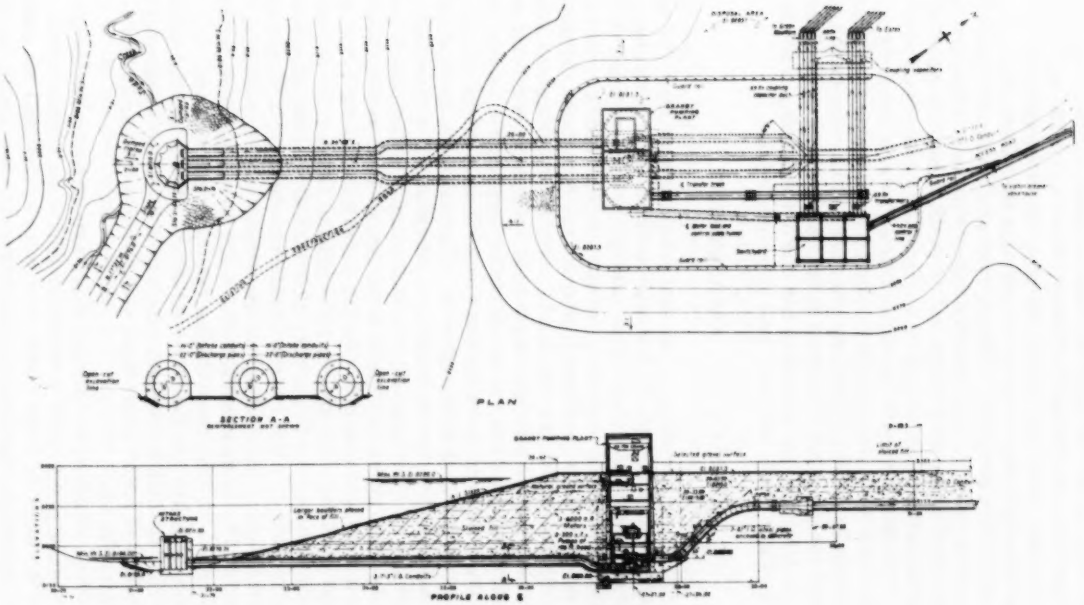


Fig. 9

The surface of the Granby Reservoir has a maximum fluctuation of 94 feet; therefore, the pumping plant has a greater vertical height than would be required in a plant of the same capacity with a normal forebay fluctuation. The final yard elevation is 22 feet above the natural ground surface and 7 feet above the maximum water surface of Granby Reservoir. The substructure extends 133 feet below the yard to provide adequate submergence for the pumps at the minimum water-surface elevation in the reservoir.

The superstructure contains a machine shop which has a 50-ton bridge crane, and control room, battery and switchgear rooms, and operator's facilities. Cable spreading, oil storage and oil purification, air compressor facilities, and unwatering pumps are on the first floor below yard level (Figure 10). The motor room is provided with a 25-ton crane for handling all equipment after it is lowered to that level by the machine shop crane. A 12-1/2-ton crane for handling the pump impeller and intermediate shaft may be moved from one pump bay to another by the motor-room crane. The sump and unwatering facilities are located in one end of the substructure and the cable shaft, elevator shaft, and stair well at the other end.

STRUCTURAL ARRANGEMENT

The superstructure presented no unusual design difficulties. The substructure offered the major design problems because of its great depth below ground. The substructure (Figure 11) is cellular in construction and consists of four main elements: the base slab, the semicircular ends or arches, the longitudinal exterior walls, and the cross walls and buttresses. (Figure 12) The cross walls and buttresses combine to serve as diaphragms or transverse plates to prevent the base slab and longitudinal walls from deflecting inward.

Early studies indicated that the semicircular ends were a natural solution for this lay-out. The hydrostatic loading places them in direct compression and thereby increases their watertightness. Further, since the loads are carried in a horizontal direction, major concentrations of moment and thrust are not introduced into the base slab, as would have been the case if buttresses had been used on the end walls. These semicircular ends were increased in thickness for a short distance above the base slab to resist the local bending stresses at the junction of the base and the arches. (Figure 13) All the floor slabs up to and including the one at elevation 8252.50 were separated from the longitudinal exterior walls and the semicircular ends by a 1-inch expansion joint. This separation insured that the exterior walls spanned in a horizontal direction. The longitudinal exterior walls were increased in thickness with depth to resist the horizontal loading acting normal to them.

The transverse plates are the major structural elements in the substructure and are 133 feet high and 84 feet wide. Each plate has an opening 50 feet wide and 33 feet high at the motor room and a second opening 28 feet wide and 11 feet high near the top.

The expense of installing an exterior 4-ply waterproofing membrane was eliminated by taking precautions to minimize shrinkage cracks. First, the base slab and the walls up to the motor floor were cooled after the concrete placement by circulating water through a system of piping similar to that used in concrete dams. (Figure 14) Second, each vertical lift was poured in four separate placements consisting of the two arches and then the two connecting longitudinal walls.

LOADS

(Figure 13) The two major loads acting upon the substructure are the dead load of the structure, and the equivalent fluid pressure from the saturated fill. The dead load of the entire structure was assumed to be distributed down through the exterior walls, the arches and the transverse plates to the foundation as a uniform load. The equivalent fluid pressure of the saturated fill was taken at 85 pounds per square foot per foot of depth. This value was used because the backfill was to be sluiced into place and would subsequently be submerged. In addition, it was felt that a relatively rapid fluctuation of the Granby Reservoir could cause a 10-foot difference in the hydraulic gradient, or saturation level, between the two longitudinal exterior walls of the plant. This gradient was assumed to vary as a straight line between the two walls.

Two minor loads were the live load and the thrust imposed by the 25° F temperature difference between the exterior members and the interior cross walls which are not in contact with the saturated backfill.

STRUCTURAL DESIGN

The design of the substructure of the pumping plant may be divided into three parts.

The first part consisted of the design of the base slab, including the vertical cantilever action of the exterior longitudinal walls and of the semicircular ends. The base slab was designed as a continuous beam spanning between cross walls and the semicircular ends. Due to the cantilever action, moments and thrusts caused by the horizontal loads were applied at the semicircular ends and the principal stresses in the base slab were computed on this basis.

In the second part of the design, the horizontal reinforcement in the longitudinal walls, cross walls, and the semicircular ends, was determined by analyzing horizontal frames cut at various heights in the structure. (Figure 11) These frames were analyzed by moment distribution, the moments being distributed into the cross walls, semicircular ends, and the longitudinal walls. The lowest horizontal frame was considered at an elevation above the base slab where the vertical cantilever action in the walls was assumed to be nonexistent. Each frame was designed to carry the full equivalent fluid pressure at that level including the effect of the 10-foot variation in the hydraulic gradient and the 25° F temperature differential between the interior and exterior members. Below the motor floor, where the horizontal loads are the greatest, the rib shortening in the end cross walls also was included because of its influence upon the moments and thrusts in the semicircular ends.

The third part of the design, the transverse frame, (Figure 13) proved to be the most complex. In the preliminary design a trial load analysis of the completed structure was made for the ultimate loading. It proved to be a valuable check for the detailed design of the transverse frame. The final analysis was performed by moment distribution and column analogy procedures. Both analyses showed that the inflection points in the buttresses at the opening for the motor room were actually inside the supports and not in the clear span. Agreement between the two methods was exceptionally good, considering the unusual proportions of the frame.

One serious problem was encountered in the cross walls that was not anticipated when the trial load analysis was requested. The final analysis showed that due to dead and live loads the cross walls below the motor floor had unexpected diagonal tensile stresses on the order of 250 pounds per square inch,

although the wall is 57 feet high and 3 feet thick. Although several relatively small openings pierce these walls, these openings did not materially contribute to the high stress. Photoelastic and stress-coat analyses of this portion of the plate verified the magnitude of the diagonal tensile stresses previously calculated. To help resist these stresses, diagonal reinforcement was added in both faces of the walls. As the compressive stresses introduced in the walls by the saturated backfill would tend to relieve these diagonal tensile forces, it was desirable that some backfill be placed before the entire structure was completed. (Figure 15) Consequently, backfilling operations were begun before concrete was placed to elevation 8272.50.

The sequence of construction of the transverse plate was of prime importance from a design standpoint as is the case in many monolithic structures that are subjected to large horizontal and vertical loads. The framing of the intermediate floors above the motor room limited the height of many of the concrete placements to approximately 10 feet. (Figure 13) A separate design was made taking into account the first placement above the motor room as the top member of the frame and the weight of the wet concrete in the placement above it acting as a dead load. Backfill was assumed to be at the level of the first placement. When the third placement above was added as dead load, the first two lifts were assumed to form the top member of a new frame and the backfill was raised to the level of the previous placement. This process was repeated until the structure and backfill were brought to grade. This method of analysis and sequence of construction resulted in the critical loading condition for the portion of the plate above the motor floor.

(Figure 16) The transition from the semicircular ends to a rectangular shape at elevation 8272.50 presented an interesting problem because the remainder of the building, 65 feet in height, is cantilevered out over the semicircular ends. To carry these superstructure walls, the basement walls from elevation 8272.50 to 8287.50 were designed as horizontal girders to carry the floor and wall loads above them.

(Figure 17) The structure was completed September 1950, and the pumps were placed in operation January 1951. The water in the reservoir has risen to elevation 8240 and the performance of the plant has been satisfactory and no leakage is evident.

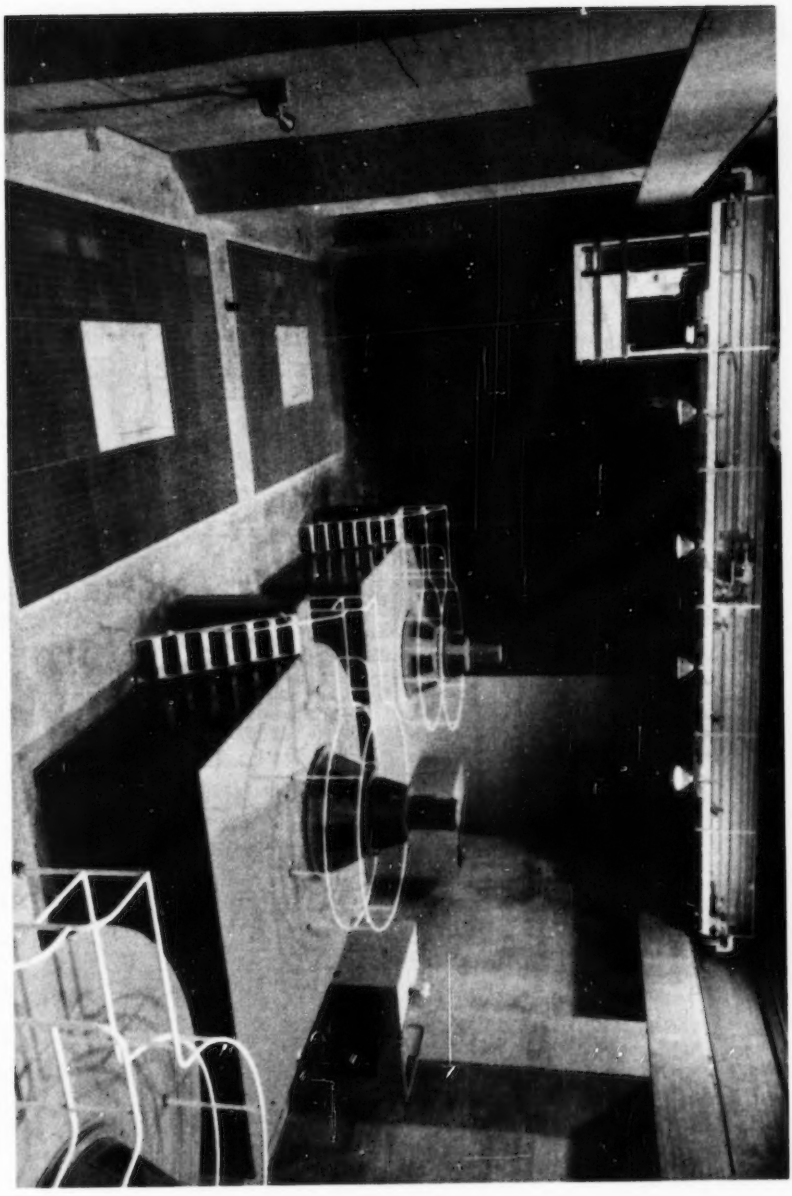
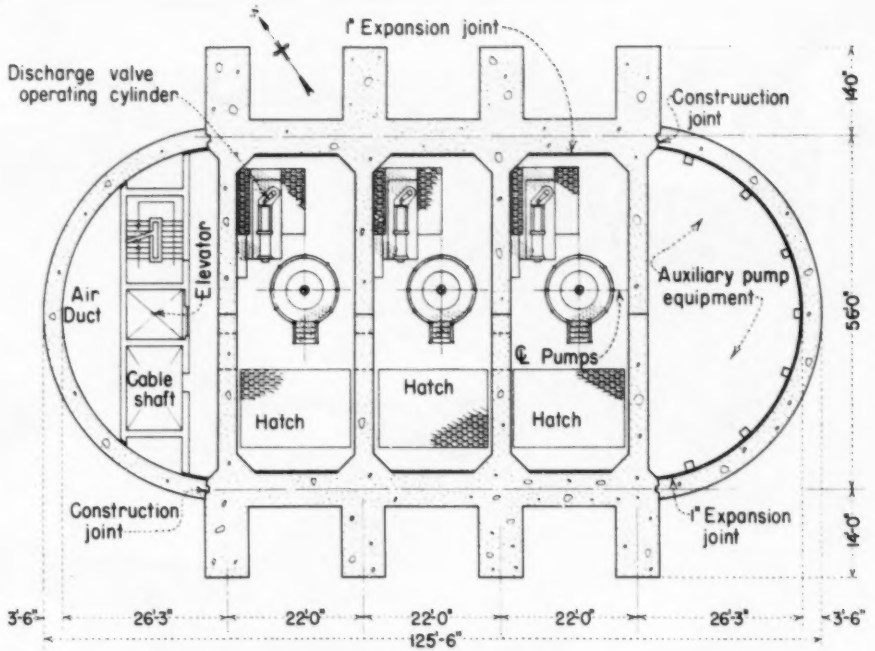
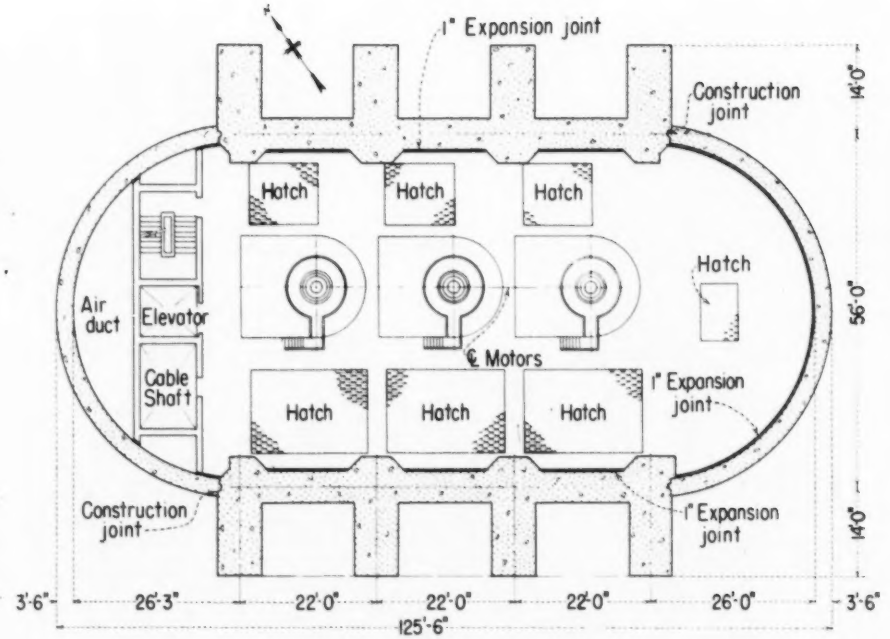


Fig. 10



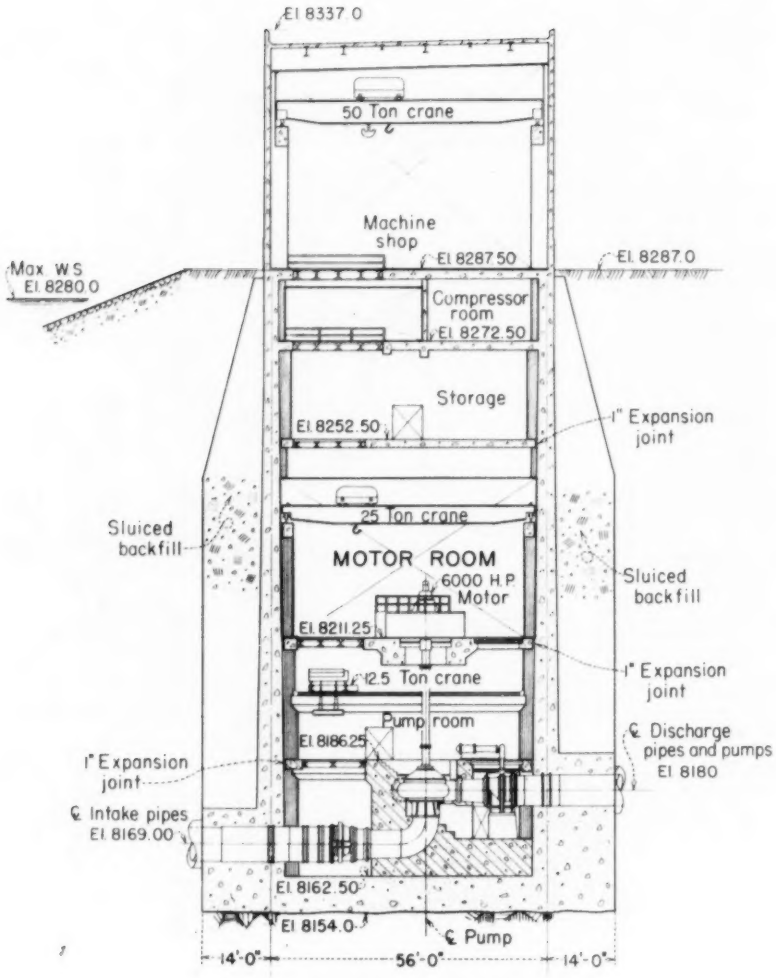
PLAN-PUMP FLOOR
EL. 8186.25

Fig. 11



PLAN-MOTOR FLOOR
EL. 8211.25

Fig. 12



TRANSVERSE SECTION

Fig. 13

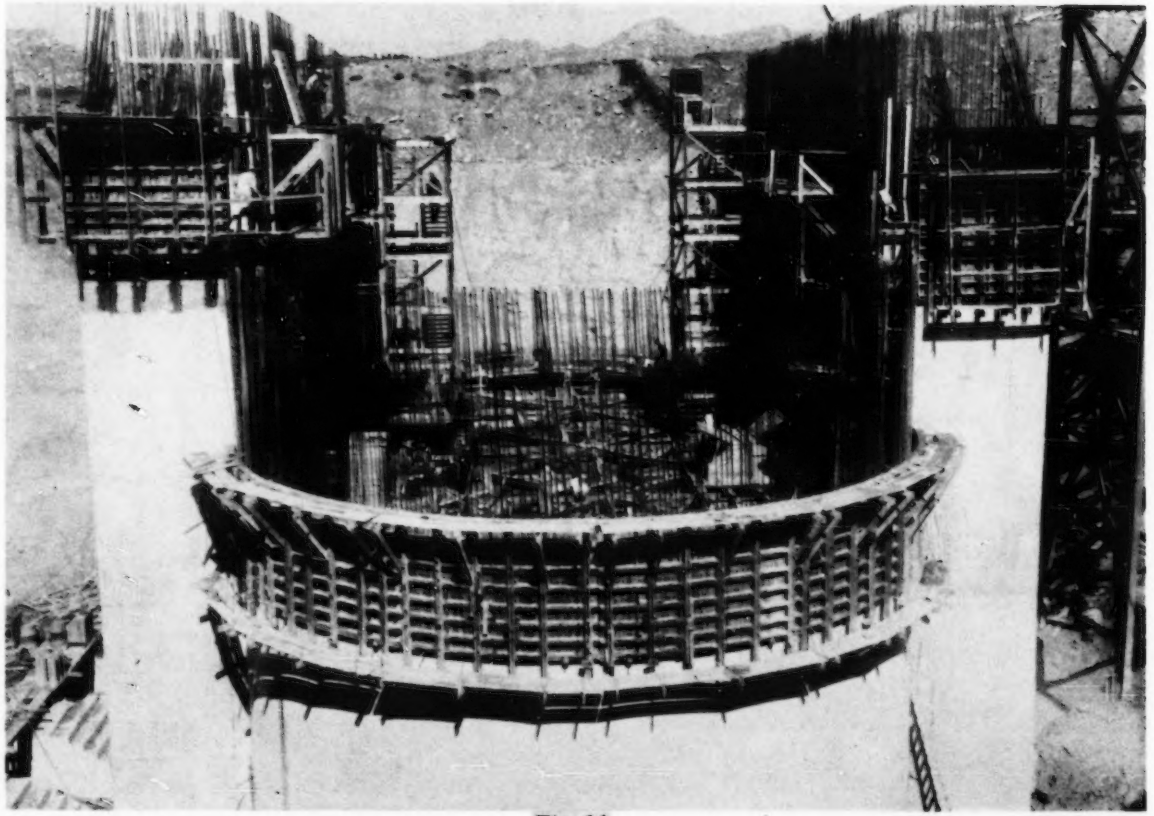


Fig. 14

Fig. 15

Fig. 16

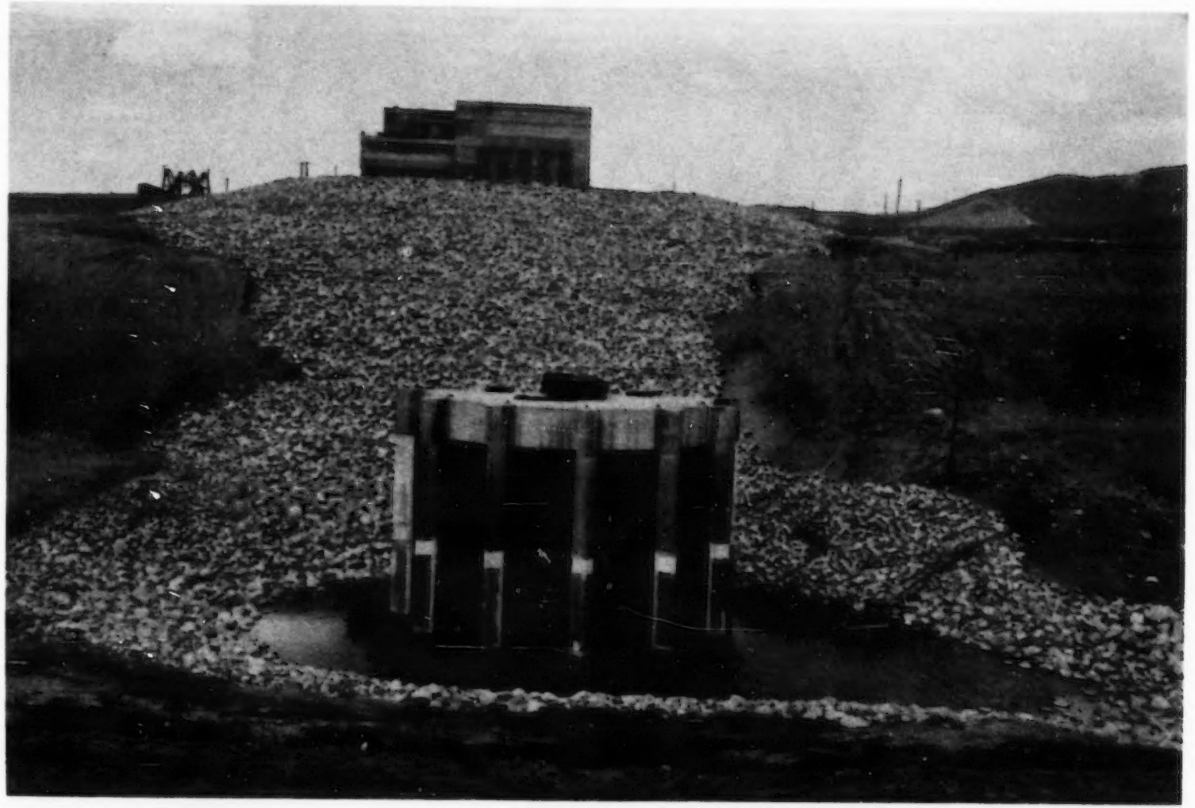


Fig. 17

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